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Seismic Fluidization and Foundation Behavior

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SYNOPSIS: Field observations of seismic settlements of foundations on granular soils due to shear flow rather than densification or liquefaction are explained in terms of the concept of seismic fluidization. The theory is briefly reviewed and used to derive seismic bearing capacity factors for shallow foundations from the standard static formulas. The reduction of bearing capacity as accelerations increase triggers incremental settlement whenever the ground acceleration exceeds some critical level whose value depends on the static design factor of safety. The total seismic settlement can be computed for a particular earthquake record by a modified sliding block approach or related to standardized incremental displacement curves for generalized earthquakes.

INTRODUCTION

Examples of foundation failures in earthquakes abound in the literature. Isolated column footings, strip footings, mat footings and even pile foundations all may fail during seismic events. Such failures are generally ascribed to liquefaction (a condition where the mean effective stress in a saturated soil reduces to zero). However, they occur even when the field conditions indicate there was only partial saturation or a dense soil and therefore liquefaction alone is a very unlikely explanation. An example of such a failure occurred during the Miyagihen-Oki earthquake of magnitude 7.8 on June 12, 1978 northeast of Sendai, Japan where the foundations of several oil storage tanks suffered from bearing capacity failure and excessive settlements (Okamoto, 1978). The subsoil for the oil storage tanks consisted of a fine sand 65m thick which had been consolidated by vibrofloatation prior to the construction of the tanks. In the United States, the settlement at the Jensen Filtration Plant during the San Fernando earthquake is a well documented example of large seismic settlement (about 100mm) experienced by a compacted material (Whitman and Bielak, 1980).

Such field observations of seismic settlements of foundations due to shear flow rather than densification or liquefaction are perhaps most easily explained in terms of the concept of "seismic fluidization" presented recently by Richards, Elms, & Budhu (1990). This theory, demonstrated by shaking-table tests in the laboratory, shows that reduction in the bearing capacity and consequent settlements should be anticipated for all types of foundations at moderate earthquake intensities even if they are on dense sand. This shear fluidization due to inertial stresses does not depend on water content and is therefore independent of any liquefaction potential (which only intensifies the phenomenon).

SEISMIC FLUIDIZATION

Inertial body forces due to vertical and horizontal accelerations $k_v g$ and $k_h g$ introduce a decrease in the effective weight $-k_v \gamma z$ and shear stresses $\tau_{xz} = k_h \gamma z$ as

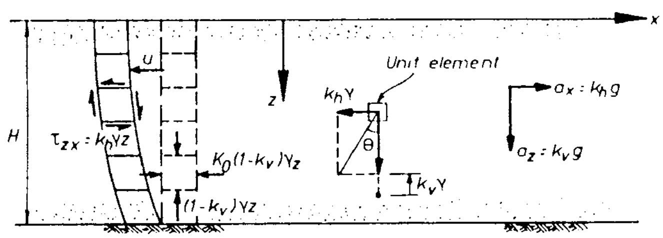


Fig. 1: Seismic Free Field

shown in Fig. 1. If we consider the Mohr-Coulomb failure criterion, these inertial stresses shift the Mohr circle, cause it to increase in radius, and

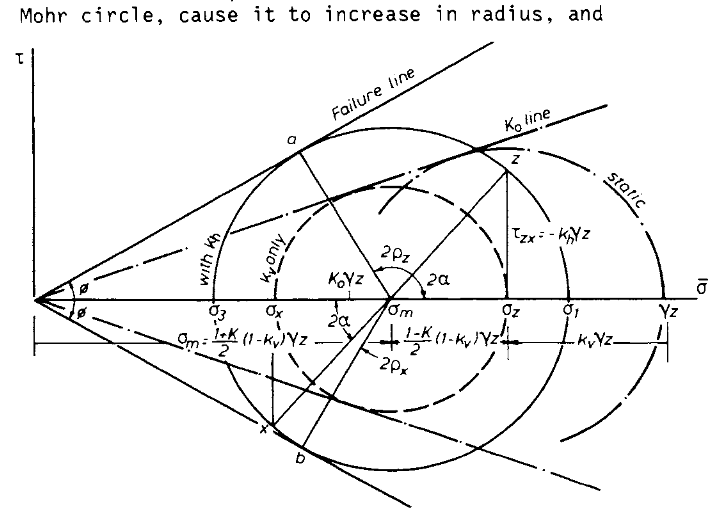


Fig. 2: Inertial "Fluidization"

rotate the principal orientation as shown in Fig. 2 for the active case. For the passive case the static circle will move to the right and enlarge rather than shrink in size and K will be considerably greater than unity.

For the active case the counterclockwise orientations of the slip surfaces develop at angles:

$$\rho_{zA} = \frac{\pi}{4} + \frac{\phi}{2} - \alpha_A \; ; \quad \rho_{xA} = \frac{\pi}{4} - \frac{\phi}{2} - \alpha_A \quad \text{.........} \quad (1)$$

inclined from the vertical and horizontal. The angle to the major principal stress orientation, α_A, is given by:

$$\tan \alpha_A = \frac{2 \tan \theta}{1 - K_{AE} + \sqrt{(1 - K_{AE})^2 + 4 \tan^2 \theta}} \quad (2)$$

in which K_{AE} is the active seismic earth pressure coefficient and

$$\tan \theta = \frac{k_h}{1-k_v}$$

where k_h and k_v are the horizontal and vertical acceleration coefficients. For the passive case the equivalent expression is

$$\tan \alpha_P = \frac{2 \tan \theta}{K_{PE} - 1 + \sqrt{(K_{PE} - 1)^2 + 4 \tan^2 \theta}} \quad (3)$$

At higher acceleration levels, more and more slip planes form until one set of slip planes is horizontal and the general fluidization state is reached. Figure 3, showing the theoretical relationship between earth pressure coefficient, effective angle of friction and acceleration ratio, presents a good summary of the theory. For a dense soil for example, with $\phi = 38°$

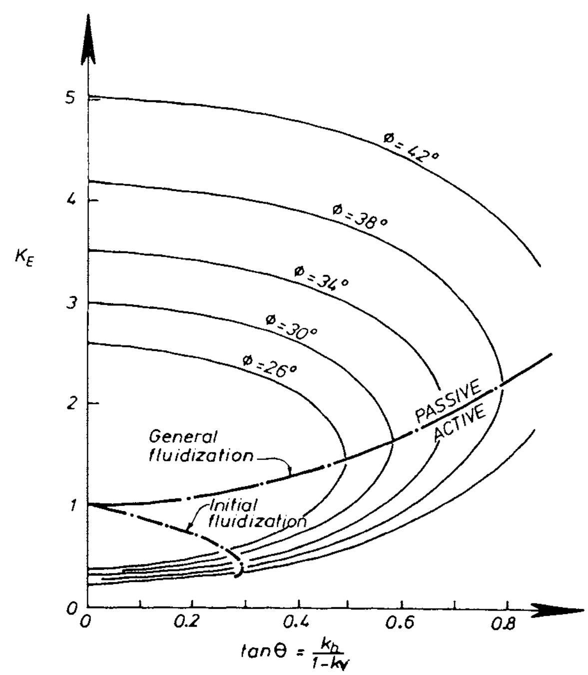


Fig. 3: Fluidization Summary

we see that, for $k_v = 0$, initial fluidization will occur at a horizontal acceleration coefficient of 0.29, and the general fluidization state is reached at $k_h = 0.78$. The actual expression for the seismic earth pressure coefficients is:

$$K_E = \frac{1 + \sin^2\phi}{\cos^2\phi} \pm \frac{2}{\cos\phi} (\tan^2\phi - \tan^2\theta)^{1/2} \quad \text{....} \quad (4)$$

where the positive sign gives K_{PE} and the negative sign gives K_{AE}.

The loss of shear strength on slip surfaces at ρ_z and ρ_x given by equations (1) with (2) or (3) leads to increased active pressures and decreased passive resistance as represented in Fig. 3. The implications for the seismic analysis and design of retaining structures is immediate and well explored. However what is often not recognized is the implication of seismic fluidization for the reduction in the bearing capacity of foundations. Even those on dry or dense soils where liquefaction cannot occur can expect a reduction in bearing capacity and consequent settlements during earthquakes. Moreover, such settlements can occur at surprisingly low seismic intensities.

Tests of a cylinder to model a circular footing

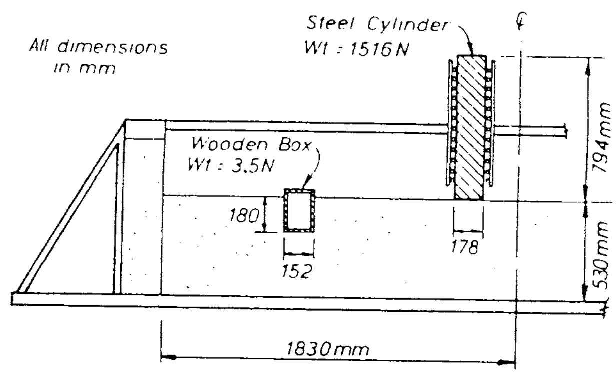
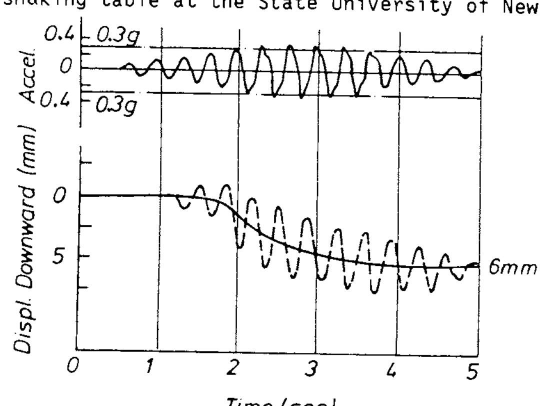
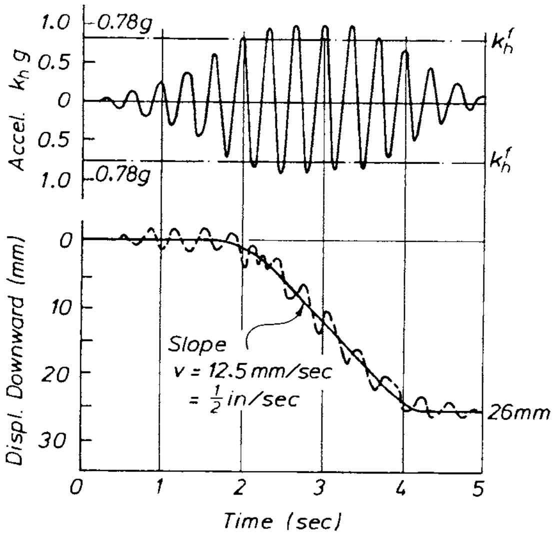


Fig. 4: Test Set Up

and also a partially-submerged buoyant box (Fig 4) verify the general behavior predicted by the theory and observed in the field during actual earthquakes. Dry, dense, Ottawa sand (ϕ=38°) was used and horizontal accelerations applied to the test box with the shaking table at the State University of New York



(a) Moderate Accelerations



(b) Strong Accelerations

Fig. 5: Seismic Behavior of a Circular Footing

at Buffalo. Fig. 5 shows the movement for the cylinder (a) at a peak horizontal acceleration of 0.3g and (b) at higher levels of acceleration $0.5 < k_h < 0.8$ where there was more general fluidization of the sand and the footing settled faster as if it were in a viscous fluid. The submerged wooden box rose in a similar fashion.

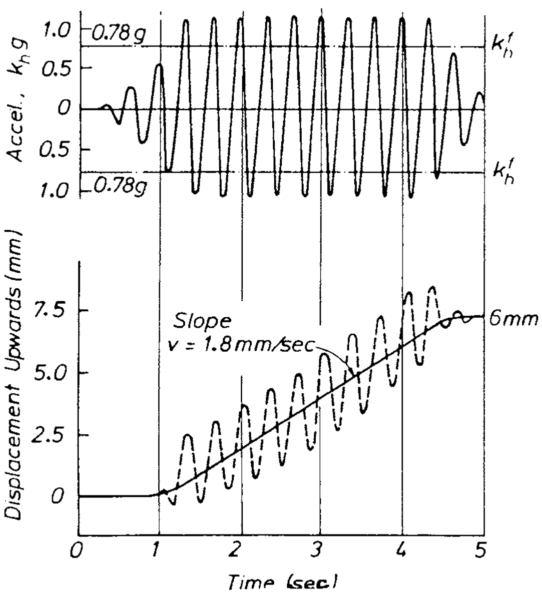


Fig. 6: Rise of the Buoyant Box with Strong Horizontal Accelerations.

SEISMIC BEARING CAPACITY AND SETTLEMENTS

Fluidization theory allows a straightforward seismic approximation to the standard static bearing capacity formulas. Both the seismic slip-line field and associated lateral pressures are now known for both the active and passive failure zones of the classic Prandtl-type mechanism. Bearing capacities can thus be calculated following standard procedures (Vesic, 1973) to generate a seismic version of the standard bearing capacity equation (Terzaghi, 1943)

$$p_{LE} = cN_{cE} + \gamma d N_{qE} + \frac{1}{2}\gamma B N_{\gamma E} \quad \text{.............. (5)}$$

where the factors N_{iE} reflect the dynamic contributions from soil cohesion c, unit weight γ, and surcharge $q=\gamma d$ as functions of ϕ and the acceleration components.

The static bearing capacity formulas for a strip footing ($L > 5B$) are (Vesic, 1973)

$$N_{qS} = \tan^2(\pi/4 + \phi/2)\, e^{\pi \tan\phi} \quad \text{................ (6a)}$$

$$N_{cS} = (N_{qS} + 1)\cot\phi \quad \text{..................... (6b)}$$

$$N_{\gamma S} = 2(N_{qS} + 1)\tan\phi \quad \text{..................... (6c)}$$

The free field solution shows that earthquake acceleration will increase K_A and decrease K_P, ρ_A and

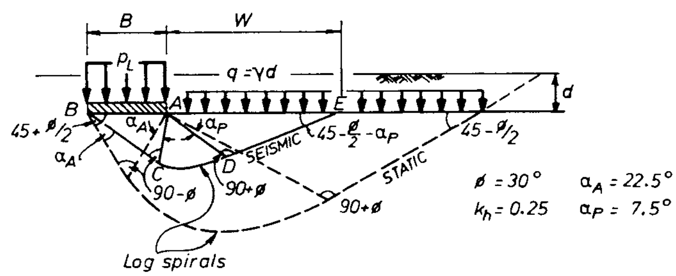


Fig. 7: Seismic and Static Mechanisms

ρ_P. Figure 7 shows how the Prandtl failure mechanism is modified by the introduction of a horizontal acceleration, assuming that the free-field solution governs the slip-surface orientations. The free field solution indicates that the active wedge beneath the footing is no longer symmetric but rotates counterclockwise through an angle α_A given by Eq. 2, though the angle at the lower point of the wedge remains constant at $90-\phi°$. The passive wedge also rotates, but this time clockwise. Once again the included angle does not change. It can be seen that the angle included in the log-spiral region must therefore shrink and become less than 90°.

A standard limit analysis (for example, Chen, 1975) leads to the expression:

$$N_{qE} = \frac{\tan(\pi/4 + \phi/2 - \alpha_A)}{\tan(\pi/4 - \phi/2 - \alpha_P)}\, e^{2(\pi/2 - \alpha_A - \alpha_P)\tan\phi} \quad \text{...(7)}$$

where the wedge rotation angles α_A and α_P are given by Eqs. 2 and 3. Note that in the derivation we are interested in ongoing rather than incipient displacement for the seismic situation. The displacements are finite, and calculable. It is important for such constant-volume displacement to use the residual or critical-state ϕ, not the peak, and to recognize that the velocity must be oriented along the slip surface boundaries, not at an angle ϕ to them.

Note also that Eq. 7 reverts to the static formulation of Eq. 6a when α_A and α_P are zero. For the limiting case of general fluidization when $\tan\theta = \tan\phi$, the slip surfaces of both active and passive wedges become horizontal, $\alpha_A = \pi/4 + \phi/2$, $\alpha_P = \pi/4 - \phi/2$ and N_{qE} becomes unity. This is to be expected as with general fluidization, the only effect of the surcharge is to act as a liquid and produce a buoyant uplift on the foundation. Finally we note that the effect of lateral inertia forces on both foundation and surcharge would be to reduce the bearing capacity still further, in the manner of the effect of an inclined load (Bolton 1979).

Assuming that Eq'ns. (6b) and (6c) hold in the dynamic situation, the seismic bearing capacity may now be calculated from Eq. 7 to give to

$$N_{cE} = (N_{qE} - 1)\cot\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

$$N_{\gamma E} = 2(N_{qE} + 1)\tan\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

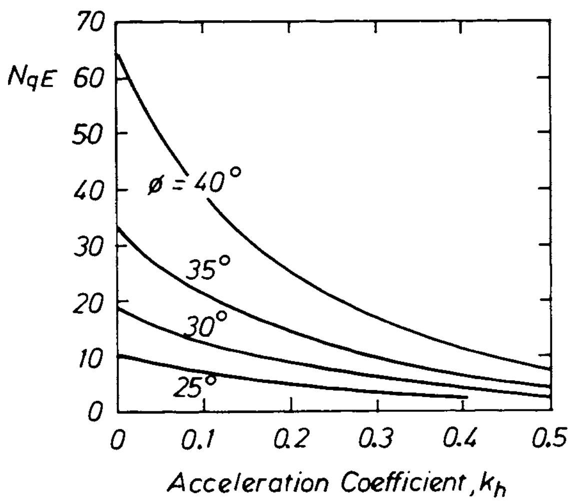


Fig. 8: Variation of Bearing Factor N_{qE} with k_h for Different ϕ

Figures 8 and 9 demonstrate the rapid deterioration in foundation strength with increasing lateral acceleration. Figure 8 gives values of N_{qE} for different values of ϕ and acceleration. As its value becomes unity with a high enough acceleration, the potential decrease in greater for higher values of ϕ. Figure 9 shows the ratio of seismic to static

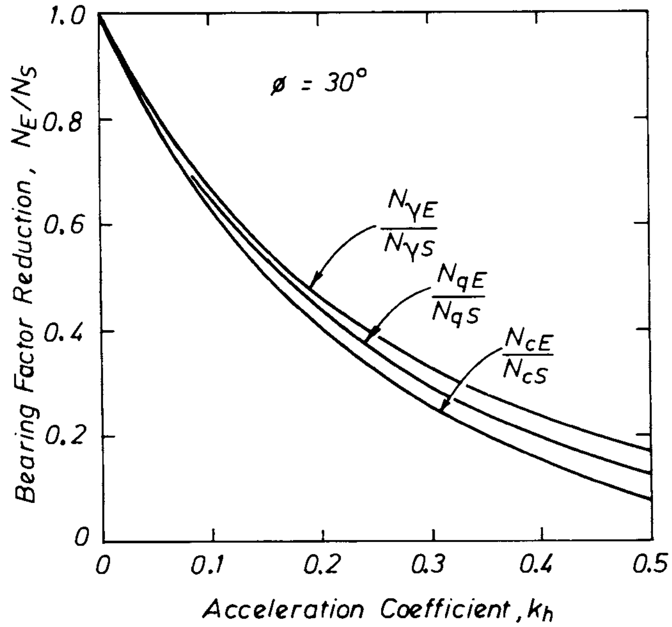


Fig. 9: Bearing Factor Reduction due to Lateral Acceleration

bearing capacity factors for $\phi = 30°$. As all decrease at very nearly the same rate we are therefore justified in writing, as an approximation,

$$p_{LE} = (N_{qE}/\, N_{qS})\, p_{LS} \qquad (10)$$

It is important to consider the implications of Fig. 9 with some care. At first sight the situation might seem more serious than it is. For example a foundation designed with a safety factor of only 2 and with $\phi = 30°$ would reach its seismic limit load at a lateral acceleration of only 0.17g; and indeed the low value of acceleration needed to reach the limit must be emphasized. However, the seismic situation cannot be looked at in the same way as the static. Reaching the seismic limit load does not mean catastrophic failure. Rather, it means that while the lateral acceleration is above the limiting value (in this case 0.17g) the foundation will settle relative to the soil with a limited velocity. Thus for each pulse above the critical value a small and finite increment of displacement will occur. The sum of such increments for an earthquake gives the total settlement to be expected.

The accululumation of settlement of the foundation can be determined from sliding block analysis similar to that used by Richards and Elms (1979) for calculating displacements of retaining walls. For a footing such as shown in Fig. 10 the settlement pattern will be asymmetric with each increment of

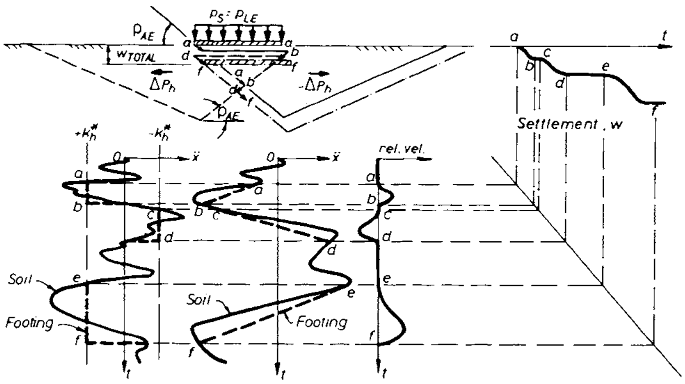


Fig. 10: Incremental Settlement by Coulomb Sliding Wedge Mechanism

settlement occurring whenever the acceleration exceeds the critical value predicted by the seismic bearing capacity equation. With this approach an engineer can, therefore, if seismic settlements cannot be prevented, adopt a displacement-control philosophy for economic design of foundations to restrict settlements in a serious earthquake to tolerable values.

CONCLUSION

A seismic limit analysis procedure for estimating seismic bearing capacity and settlement by the classical upper-bound approach has been outlined and explored in some detail. This Prandtl-type mechanism is not claimed to be precise but it does allow the straightforward development of seismic bearing capacity factors directly related to their static counterpart. The comparison of the two depicts clearly the rapid deterioration of foundation strength with increasing acceleration. This, in turn, explains observations both in the field and in the laboratory of seismic bearing failures and excessive settlements which are not attributable to either liquefaction or a dynamic increase in load.

Thus while many aspects of this particular solution must be refined it can serve now to give greater fundamental insight into this aspect of earthquake engineering so as to develop better procedures for the design of foundations in seismic zones.

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